

A simplified method in comparison with comprehensive interaction incremental dynamic analysis to assess seismic performance of jacket-type offshore platforms

M. R. Zolfaghari¹ · A. Ajamy¹ · B. Asgarian¹

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Abstract The primary goal of seismic reassessment procedures in oil platform codes is to determine the reliability of a platform under extreme earthquake loading. Therefore, in this paper, a simplified method is proposed to assess seismic performance of existing jacket-type offshore platforms (JTOP) in regions ranging from near-elastic to global collapse. The simplified method curve exploits well agreement between static pushover (SPO) curve and the entire summarized interaction incremental dynamic analysis (CI-IDA) curve of the platform. Although the CI-IDA method offers better understanding and better modelling of the phenomenon, it is a time-consuming and challenging task. To overcome the challenges, the simplified procedure, a fast and accurate approach, is introduced based on SPO analysis. Then, an existing JTOP in the Persian Gulf is presented to illustrate the procedure, and finally a comparison is made between the simplified method and CI-IDA results. The simplified method is very informative and practical for current engineering purposes. It is able to predict seismic performance elasticity to global dynamic instability with reasonable accuracy and little computational effort.

Keywords Uncertainty · Ground motion records · Comprehensive interaction incremental dynamic analysis (CI-IDA) · Static pushover analysis (SPO) · Jacket-type offshore platform

Introduction

The levels of damage resulted from some major events such as the San Fernando (1971), Loma Prieta (1989) and Northridge (1994) earthquakes indicate that it is essential to modify the building codes to improve seismic resistance in structures. Therefore, the Performance-Based Earthquake Engineering (PBEE) methodology has been applied in design and construction from 2000 to evaluate the seismic performance of existing structures under extreme loads (Krawinkler and Miranda 2004). While the ever-increasing processing power of computers has improved upon analytical applications of structural models, the analyses have been conducted from elastic static analyses to dynamic elastic, nonlinear static and finally nonlinear dynamic analyses.

The nonlinear static analysis normally called pushover analysis is a procedure in which a structural model is subjected to a predetermined lateral load pattern and demonstrates the relative inertia forces generated at locations of substantial mass. In this procedure, the intensity of the load increases (the structure is ‘pushed’) and incremental process continues until a predetermined displacement is achieved. Among several pushover analyses, the static pushover (SPO) analysis has no strict theoretical base such that the response of the structure is controlled by the first mode of vibration through introducing a constant load pattern. Moreover, the principle objective of SPO is based on structural static theory and it does not correctly predict the dynamic failure modes (Krawinkler 2001).

✉ A. Ajamy
azadeh_ajamy@yahoo.com; a_ajami@dena.kntu.ac.ir
M. R. Zolfaghari
mzolfaghari@kntu.ac.ir
B. Asgarian
asgarian@kntu.ac.ir

¹ Department of Civil Engineering, K.N. Toosi University of Technology, No. 1346, Valiasr St., Mirdamad Intersection 19967, P.O.Box. 15875-4416, Tehran, Iran

Since the result of the static pushover provides information that is not obtained from an elastic static and dynamic analysis, it is one of the most important methods to offer seismic performance. The following are the advantages of SPO analysis (Govind et al. 2014):

1. The method shows overall structural behaviours and their distributions along the height.
2. It provides the sequential formation in the individual structural components, and also identifies location of weak points or potential failure modes in the structure.
3. It shows consequences of strength deterioration on the behaviour of structural system in both plan and elevation.

In addition, there are some limitations in the SPO analysis described in the following (Khan and Vyawahare 2013):

1. This method represents inaccurate responses in which higher mode effects are significant.
2. It will be necessary to perform the analysis with displacement rather than force control.
3. SPO analysis neglects duration effects, number of stress reversals and cumulative energy dissipation demand in the structures.
4. This method cannot consider the progressive changes in modal properties.
5. Finally, the SPO method is able to detect only the first local mechanism.

Chopra and Goel (2001) suggested the modal pushover analysis (MPA) method based on structural dynamics theory. While the MPA method provides sufficient accuracy in the estimation of seismic demands, it is conceptually simple and straightforward (Chopra and Goel 2001). Moreover, Chopra et al. (2004) proposed the modified modal pushover analysis method (MMPA) approach assuming higher modes result in only elastic behaviour of the structure (Chopra et al. 2004). Since the MMPA method neglects the variability of higher modes that can lead to nonlinear behaviours stage in structures, it cannot always predict accurately structural behaviours (Themelis 2008).

To remedy this deficiency in the procedure, Vamvatsikos (2002) proposed the incremental dynamic analysis (IDA) method. This approach is an emerging structural analysis method performing a series of nonlinear dynamic analyses under a suite of multiply scaled ground motion records. The method is usually very complex and time consuming because of the complex nature of strong ground motions.

To reassess the jacket offshore platforms, there are two general forms: (1) static pushover analysis (2) time domain analysis. Asgarian and Ajamy (2010) applied the IDA

method for new designed jacket-type offshore platforms in the Persian Gulf and defined different structural limit states. Also, Asgarian and Rahman Shokrgozar (2013) evaluated the seismic performance of an existing jacket-type offshore platform with float-over deck using a probabilistic method. They estimated the mean annual frequency and confidence levels in this type of structures.

Once the IDA procedure is applied to perform nonlinear dynamic analyses, aleatory (record-to-record) uncertainty only is considered. Obviously, the best prediction will be achieved when the effects of epistemic uncertainty are also considered. Therefore, Cornell et al. (2002) and Ellingwood (2007) proposed one approach to combine the effects of different sources of uncertainties in terms of the confidence interval approach. The distribution associated with epistemic uncertainty in this case may be obtained from the first-order-second-moment reliability, Monte Carlo Simulation (MCS) methods or expert judgment.

Liel (2008) used the response surface method as a functional relationship between the input random variables and limit state criterion to find collapse capacity limit. She assessed collapse risk of reinforced concrete moment frame buildings. Moreover, the MCS method along with response surface was applied to propagate modelling uncertainties and fit a response surface to sensitivity analysis results. Furthermore, Liel (2008) have shown that in very nonlinear structural responses, the effect of assumed epistemic uncertainty can be more significant in the assessment.

In oil and gas industry, the effects of epistemic uncertainty have been studied by Ajamy et al. (2014) in an existing jacket-type offshore platform. They proposed a new method named the Comprehensive interaction incremental dynamic analysis (CI-IDA) method to propagate epistemic uncertainties associated with aleatory uncertainty in different parts of structural systems such as surrounding soil, pile, structural elements and finally ground motion records.

Since accurate estimates of the seismic performance require performing the costly analyses, a simplified method is introduced in this paper. It creates a direct connection between static pushover analysis and time domain analyses. Then, the sufficiency of the simplified method is examined in an existing jacket-type offshore platform in the Persian Gulf using comparison with CI-IDA results.

Requirements to perform the simplified method

The simplified procedure includes three steps: (1) to create a structural model to offer realistic situations (2) to perform SPO analysis and generate SPO curves in different levels (3) to calculate different slopes in the SPO curve.



Jacket-type offshore platform modelling

Since oil and gas industry relies on offshore platforms, the reassessment of jacket-type offshore platforms under extreme loads is of significant importance. In this paper, one of the existing jacket-type offshore platforms constructed in the early 1970s has been chosen to reassess seismic performance.

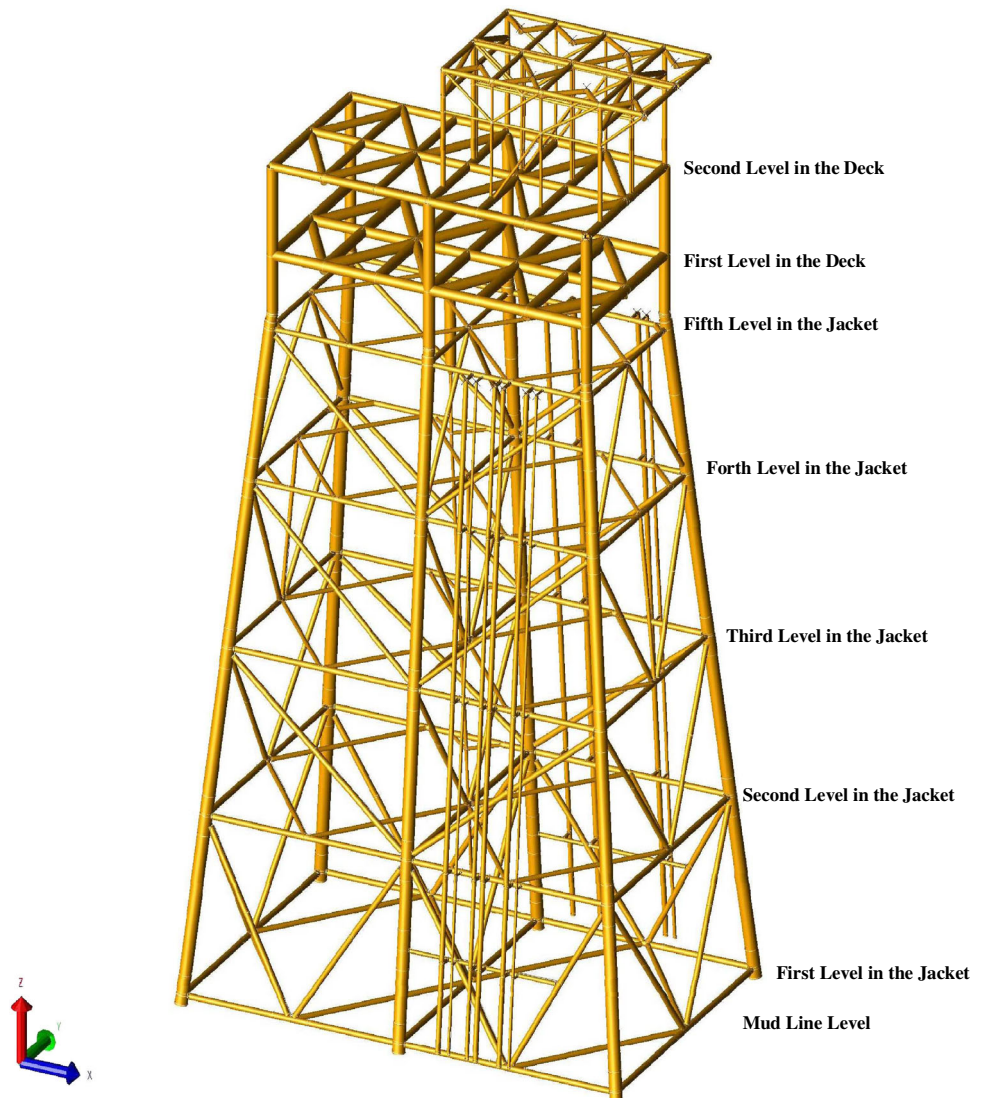
The platform is in the Persian Gulf, approximately 100 km southwest of the Kharg Island export terminal. The field is on the Iran-Saudi Arabian water border, with the Saudi-Arabian portion of the field being called the Marjan field.

Figure 1 presents the two-dimensional model of the selected offshore platform. As shown in Fig. 1, the platform is a six-legged platform with 75 m height. To illustrate the methodology, a two-dimensional model associated with its deep piles was created in the OpenSees (2006) analysis platform.

The main structural subsystems of the model consisted of 42 frame elements which include 3 jacket leg members, 25 beam members, 14 diagonal brace members. The platform geometry includes the total length and the number of spans of the jacket and the deck, the cross-sectional dimensions of the piles, legs, beams and braces and support details to model boundary conditions. Moreover, the model entails conductors and risers and their dimensions are based on the existing data in the drawings.

In simulation part, all deck members were modelled by elastic members and a `forceBeamColumn` element object was used to simulate the jacket members. The object is based on the iterative force-based formulation and addresses the nonlinear behaviours in terms of both distributed plasticity and plastic hinge integration. Moreover, to simulate pile elements in OpenSees software, a `dispBeamColumn` element object which is a distributed-plasticity and displacement-based beam-column element was used.

Fig. 1 A view of the two-dimensional model configuration



The cross-sections of the elements were modelled using the fibre element to model the nonlinear behaviours of the platform. In this research, the cross-sectional geometric properties of tubular members consist of outer diameter and thickness as exists in the drawings. The application of fibre elements is discussed in more detail elsewhere (Asgarian et al. 2005, 2006) to address buckling and post-buckling behaviours of tubular members and also nonlinear analyses of platforms.

To create an exact geometric transformation, Corotational Transformation was applied in the jacket elements. It uses beam stiffness and resisting force in the basic system to convert the global coordinate system. In piles, P-Delta Transformation was used to consider the geometric nonlinearities and large deformation effects in the nonlinear time history analyses.

The mass of the platform consists of the distributed mass of piles, legs, beams and braces considered in the length of modelled elements. Based on the reported masses in the in-place analysis report, the mass of various appurtenances such as conductors, risers, mud mats, boat landings, walkways, and caissons is added to the mass of the electrical, mechanical and structural parts; then, it is considered as lumped mass at the jacket structural nodes. In this study, the presence of water is neglected and the effect of water–soil–pile–structure interaction is not addressed while the model includes flooded members.

Since structure, pile and surrounding soil interact with each other under static and dynamic loading, their interaction is complex and requires a comprehensive understanding. On the other hand, this issue plays a significant role in determining the response of structures such as offshore platforms during an earthquake. For instance, seismic soil–pile–structure interaction (SSPSI) usually increases the periods of the system or varies the system damping (Azarbakht et al. 2008).

The several researchers developed numerous simplified methods to address the effects of SSPSI. Among these models, those based on the static/dynamic beam-on-nonlinear-Winkler-foundation (BNWF) method, often referred to as the p–y method, are commonly utilized to simulate SSPSI problems (Winkler 1867).

In this study, the BNWF method was used to model the effects of SSPI. In this method, independent horizontal and vertical nonlinear springs have been applied along the pile shaft to represent thicknesses, stiffness, and damping characteristics of each layer. The layers (39 layers) were modelled using p–y, t–z and q–z elements according to API-RP2A (1993) provisions and then attached to pile nodes through zero-length elements.

In the analysis part, the gravity analysis is carried out using Newton algorithm. The algorithm solves the nonlinear equations and is able to be updated each iteration.

Moreover, the Norm Displacement Increment test command was used to construct a Convergence test object. The command determines if convergence has been achieved at the end of an iteration step.

In time history analyses, the earthquake excitation is simulated using MultipleSupport Excitation pattern. It constructs an ImposedMotionSP constraint used to enforce the response of a degree-of-freedom at a node in the model. The response is obtained from the GroundMotion object associated with the constraint. In the material part, the uniaxialMaterial Steel01 command was used to consider material properties. It applies to construct a uniaxial bilinear steel material object with kinematic hardening and optional isotropic hardening described by a nonlinear evolution equation.

Since the dynamic characteristics of the offshore platform are explicitly portrayed through modal analysis, it was carried out in OpenSees Post Processor (OSP) before each transient analysis to represent natural frequencies. Then, the results were compared with the periods computed during the assessment stage using the Structural Analysis Computer System (SACS) software.

The frequencies and the mode shapes assumed by the platform are determined analytically based on the stiffness, mass, and damping properties of the simulated system. The results in the OSP software match well with the SACS results. For instance, the first natural period is equal to 2.17 s in OSP while it is equal to 2.25 s in SACS. Also, the mode shape of the periods corresponds to motion in the transverse direction of the platform.

The modal results, specifically modal periods, are the main parameters used in time history analyses. Moreover, geometric nonlinearity was included in all dynamic analyses and damping was also considered using Rayleigh damping. Since the transient analysis was often unable to converge to a solution, a solution procedure script was developed within the analysis file that tried a number of different solution algorithms, time steps, and convergence criterion until a solution could be achieved.

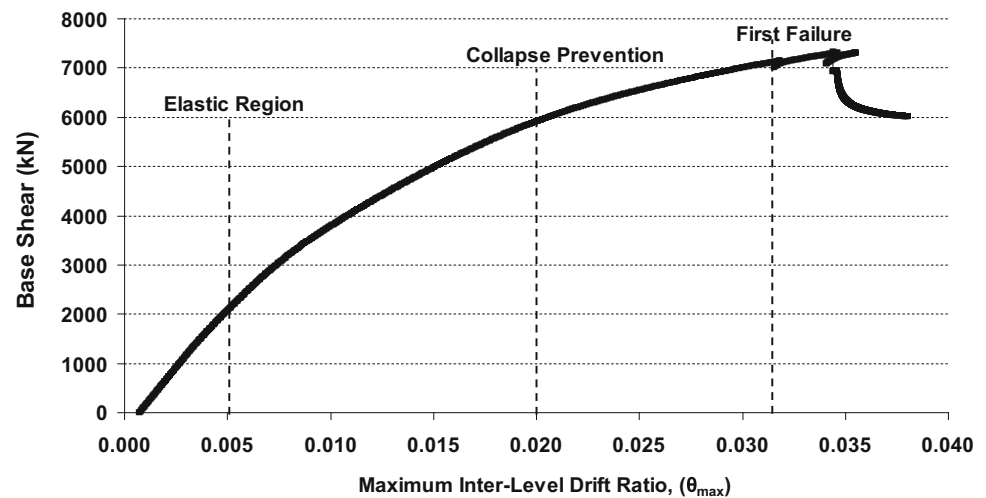
Static pushover analysis

SPO analysis is applied to establish the static ultimate strength of the platform and the failure sequence for the selected loading pattern. Moreover, it is utilized to monitor seismic performances of the platform.

In this study, Fig. 2 shows the variation of base shear versus maximum inter-level drift in the SPO curve at the mud line level of the offshore platform. Based on FEMA 350 (2000a, b), ASCE 41-06 (2007) and engineering judgment, the collapse prevention (CP) is considered by the value of $\theta_{\max} = 2\%$ in jacket-type offshore platforms. In



Fig. 2 The static pushover curve at the mud line level



the Fig., there are three regions including the elastic linear, collapse prevention and first failure regions in the SPO curve.

Since the SPO curve is usually based on base shear versus θ_{\max} coordinates, it needs to be transformed into IM and DM axes. Therefore, it is necessary to divide the base shear by the total structural mass by times a proper factor to convert it to appropriate intensity measure. Figure 3 represents the SPO curves at the working point and mud line levels of the jacket and the top level of the deck. They show the variability of θ_{\max} versus $S_a(T_1, 5\%)$ axes to offer the damage levels in the platform. It is obvious that the most vulnerable level is the mud line level.

The simplified method using SPO analysis

To introduce the simplified approach, it is necessary to obtain the slopes of the SPO curve in different parts. Figure 4 represents the SPO curve at the mud line level based on DM versus IM axes. As illustrated in the Fig., since there are three separate regions in the SPO curve, it is necessary to obtain three different slopes to accomplish the simplified method.

At the beginning, to define the elastic linear region, the elastic segment of the simplified curve coincides the elastic slope of the SPO curve; then, the effect of uncertainty is gradually added towards the elastic slope in the SPO curve. In this place, the point A is obtained. The effect of uncertainty varies between 0.35 and 0.50 at the mud line level (Ajamy et al. 2014).

Veletsos and Newmark (1960) reported that moderate period structures follow “equal-displacement” rule in non-negative stiffness region. Therefore, in the second part, the slope of this region in the simplified method corresponds to

a continuation of the elastic region in the simplified curve (the point B). The second part ends at the CP region.

Finally, to define the third part in the simplified curve, the effect of uncertainty is added the slope of the SPO curve in negative region. It should be mentioned that the slope decreases in the SPO curve. This part starts from the CP region and ends at the first failure region (the point C).

The aforementioned steps have been illustrated in Fig. 4 from the results of SPO curve. In the Fig., the dashed line shows the simplified curve and the solid line shows the SPO curve at the mud line level. Since the effects of uncertainty are considered in the simplified method, it is necessary to validate the results with the CI-IDA method.

Fundamentals of the CI-IDA method

The CI-IDA method involves three following steps: (1) to simulate structural models to present realistic modelling case, (2) to identify uncertainties in terms of aleatory and epistemic uncertainties, (3) to carry out nonlinear dynamic analyses subjected to ground motion records and to provide the relationship between damage measure (DM) and intensity measure (IM) using the curves of single-CI-IDAs and multi-record CI-IDAs and their summary.

Uncertainty considered in this study

In this paper, it is assumed independence between the aleatory and epistemic random variables, as illustrated in Fig. 5. Epistemic uncertainties are defined in different parts of the structural system including pile, structural elements and surrounding soil and record-to-record variability in actual ground motion records is treated as aleatory uncertainty.



Fig. 3 The SPO curves at the working point and mud line levels of the jacket and 2nd level of the deck

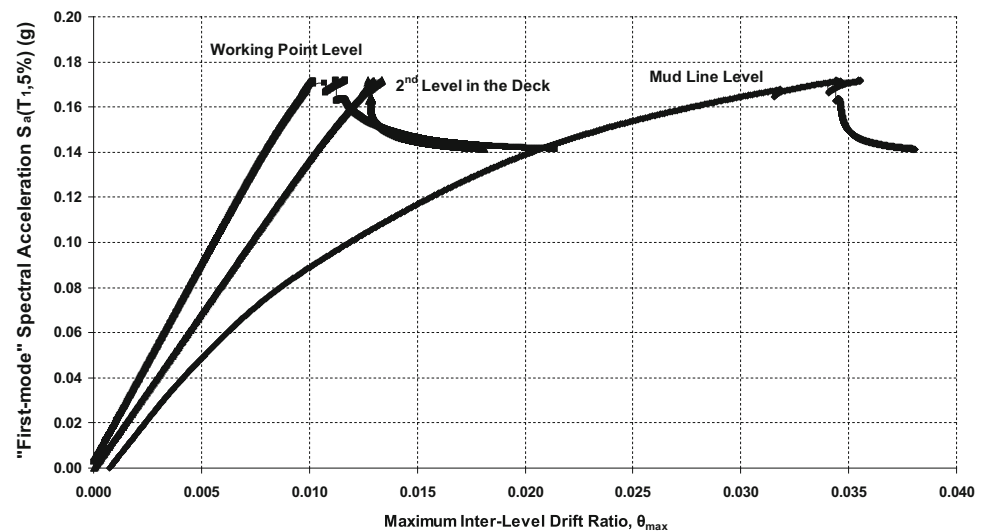
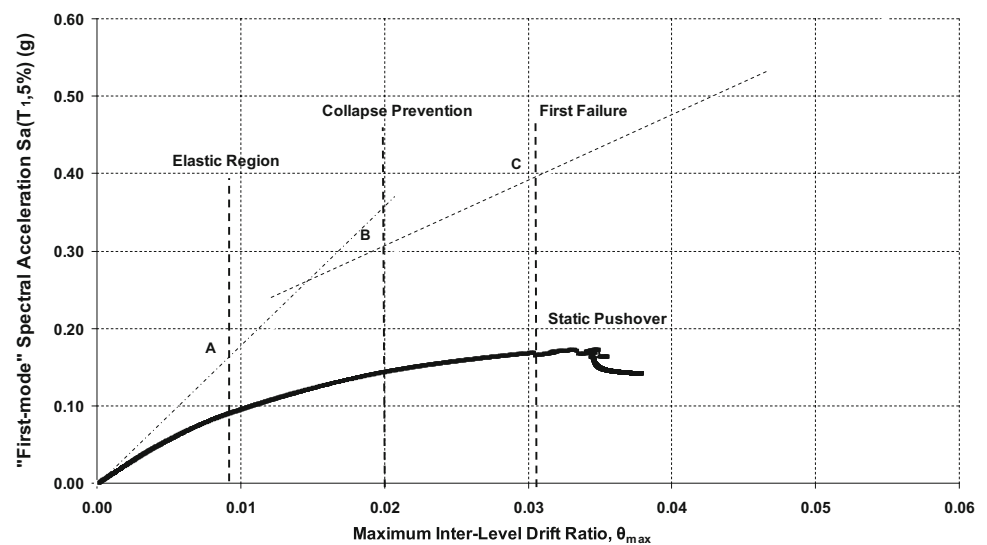


Fig. 4 The simplified curve (dashed line) and the SPO curve (solid line) at the mud line level



In the structural simulation section, yield stress of legs and braces and modulus of elasticity and in the soil simulation section, shear-wave velocity, shear modulus reduction and damping ratio are considered as epistemic uncertainty. Table 1 shows statistical data of the structural and soil properties in different parts including the used symbol, the type of distribution function, mean values of the variables, their coefficient of variation (COV) and finally the used references.

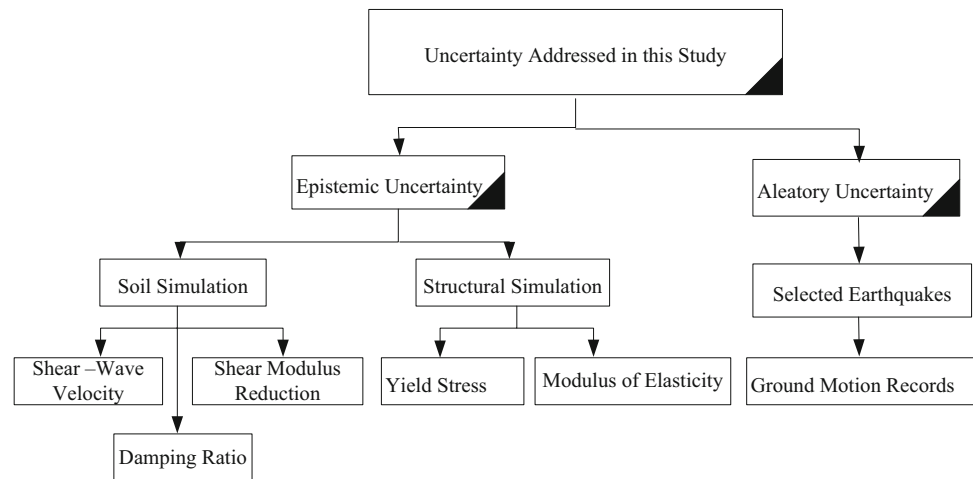
The technique of latin hypercube sampling (LHS) in conjunction with simulated annealing (SA) optimization (Vorechovsky 2004) was applied to propagate the effects of epistemic uncertainties in the model. The procedure is described in many references elsewhere (Vorechovsky 2004, 2012; Vorechovsky and Novak 2009) and only a brief summary is provided here.

In this technique, a difference matrix named the error correlation matrix (E) is determined according to Eq. 1. It is the difference between the target correlation matrix (T) and the actual correlation matrix (A). T is introduced by user and A can be estimated by Pearson's correlation coefficient, Spearman's rho or Kendall's tau (Vorechovsky and Novak 2009)

$$E = T - A. \quad (1)$$

In the equation, the number of the input random variables is equal to five (based on the Table 1) and the number of simulations is assumed equal to 15 (Vorechovsky 2012). After the application of LHS associated with SA, the following matrix (2) is presented in which the lower triangle is equal to the target correlation matrix T (T) and the upper triangle is equal to the Spearman rank order correlation



Fig. 5 Uncertainty addressed in this Study**Table 1** Statistical data of the structural and soil properties

Random variables	Symbol	Type	Mean	COV	References
Parameters influencing uncertainties in structural element model					
Yield stress of legs	$f_{Y,L}$	Lognormal	360 Mpa, 355 Mpa	0.07	Haselton (2006); JCSS (2001)
Yield stress of braces	$f_{Y,B}$	Lognormal	360 Mpa, 355 Mpa	0.07	Haselton (2006); JCSS (2001)
Modulus of elasticity	E_S	Lognormal	2.00×10^5 Mpa	0.03	Haselton (2006); JCSS (2001)
Parameters influencing uncertainties in surrounding soil and pile element model					
Shear-wave velocity	V_S	Lognormal	Computed	Computed	Toro (1995)
Shear modulus reduction	G/G_{\max}	Normal	Computed	Computed	Darendeli and Stokoe (2001)
Damping ratio	D	Normal	Computed	Computed	Darendeli and Stokoe (2001)

matrix A (A). The correlation coefficients of the matrix reflect the studied epistemic uncertainties in each simulation

$$\begin{pmatrix}
 f_y & E_S & V_S & \frac{G}{G_{\max}} & D \\
 1 & -0.00714 & 0.01071 & 0.00000 & -0.01429 \\
 0 & 1 & -0.02500 & 0.00714 & -0.00714 \\
 0 & 0 & 1 & -0.00714 & 0.00357 \\
 0 & 0 & 0 & 1 & -0.48929 \\
 0 & 0 & 0 & -0.5 & 1
 \end{pmatrix} \begin{matrix} f_y \\ E_S \\ V_S \\ \frac{G}{G_{\max}} \\ D \end{matrix} \quad (2)$$

To address record-to-record variability according to Table 2, a set of 10 ground motion records is enough to be used to offer proper accuracy in the estimation of structural behaviours. There are two main factors affecting the record selection process. One of them is the soil type and the other is the average fundamental period of the studied structure (Shome and Cornell 1999; Azarbakht and Dolšek 2007).

The ground motion records belong to the far-field ground motions from the PEER (2006) database. Based on the local geotechnical report, the site class D is considered according to recent NEHRP (2001) seismic provisions.

Also, it includes large source-to-site distances between 20 and 70 km with relatively large magnitudes of 6.0–7.3, and reverse or reverse-oblique faulting mechanisms.

In addition to the above criteria, Arias intensity was used to select the records. Arias intensity is a ground motion parameter that captures the potential destructiveness of an earthquake as the integral of the square of the acceleration time history. It correlates well with several commonly used demand measures of structural performance, liquefaction, and seismic slope stability (Travasarou et al. 2003). Using site-specific hazard curve, spectral accelerations for 2 and 10 % probability of exceedance in 50 years are equal to 0.26 and 0.16 g, respectively (Ajamy et al. 2014).

After the record selection process, each record was scaled in ten levels through the hunt and fill algorithm (Vamvatsikos and Cornell 2004). To complete the requirements in CI-IDA, the 5 %-damped first-mode spectral acceleration of the platform ($S_a(T_1, 5\%)$) and maximum inter-level drift ratio (θ_{\max}) were chosen as a relative efficient IM and DM, respectively.

Since the seismic response of the surrounding soil strongly affects the structural performance, equivalent-



Table 2 The set of ten ground motion records applied

No.	Station	Component	Mechanism	PGA
1	Parkfield—Fault Zone 14 in Coalinga-01	090	Reverse-oblique	0.274
2	Slack Canyon in Coalinga-01	045	Reverse-oblique	0.166
3	Parkfield—Fault Zone 16 in Coalinga-01	000	Reverse-oblique	0.195
4	25 SMART1 C00 in Taiwan SMART1	000	Reverse-oblique	0.172
5	APEEL 7—Pulgas in Loma Prieta	000	Reverse-oblique	0.156
6	Hayward—BART Sta in Loma Prieta	310	Reverse-oblique	0.156
7	Hollister Diff. Array in Loma Prieta	165	Reverse-oblique	0.269
8	Palo Alto—SLAC Lab in Loma Prieta	360	Reverse-oblique	0.278
9	LA—Univ. Hospital in Northridge-01	095	Reverse	0.214
10	Lake Hughes #12A in Northridge-01	090	Reverse	0.174

linear site response analyses are used to simulate the effects of the ground response using one-dimensional (1D) models. The models assume that seismic waves propagate vertically through horizontal sediment layers. Among several computer programs, the DEEPSOIL software was selected to perform the seismic site response analysis (Hashash et al. 2012).

According to Table 1, treatment of uncertainties is considered in the input ground motions, the nonlinear properties of the surrounding soil and the shear-wave velocity of the site (Rathje et al. 2010). DEEPSOIL uses outcropping motions as the input ground motions, and then the nonlinear properties and the shear-wave velocity are defined in the calculation part by user.

To simulate the shear-wave velocity, the statistical models of Toro (1995) were used in which a lognormal distribution at mid-depth of the layer and an interlayer correlation are applied. Equation (2) predicts the shear-wave velocity in the i th layer ($V_s(i)$).

$$V_s(i) = \exp\{\ln[V_{\text{median}}(i)] + Z_i \cdot \sigma_{\ln V_s}\} \quad (3)$$

where $V_{\text{median}}(i)$ is the mean shear-wave velocity at the mid-depth of the layer, $\sigma_{\ln V_s}$ is the standard deviation of the natural logarithm of $V_s(i)$, Z_i is the standard normal variable of the i th layer. All random variables are generated based on the site class and defined in more detail elsewhere (Toro 1995).

To model the nonlinear soil property, the empirical models of Darendeli and Stokoe were used (JCSS 2001). In these models, the mean variation of the nonlinear soil properties follows a normal distribution; the standard deviation of the normalized shear modulus (σ_{NG}) and the standard deviation of the damping ratio (σ_D) are predicted by Eqs. (4, 5).

$$\sigma_{NG} = 0.015 + 0.16 \sqrt{0.25 - (G/G_{\max} - 0.5)^2} \quad (4)$$

$$\sigma_D = 0.0067 + 0.78 \sqrt{D(\%)} \quad (5)$$

The variation of Eqs. 3 and 4 is described in detail elsewhere (33). Since G/G_{\max} and D curves are not independent ($\rho_{D,NG} < 0$), they are correlated G/G_{\max} and D

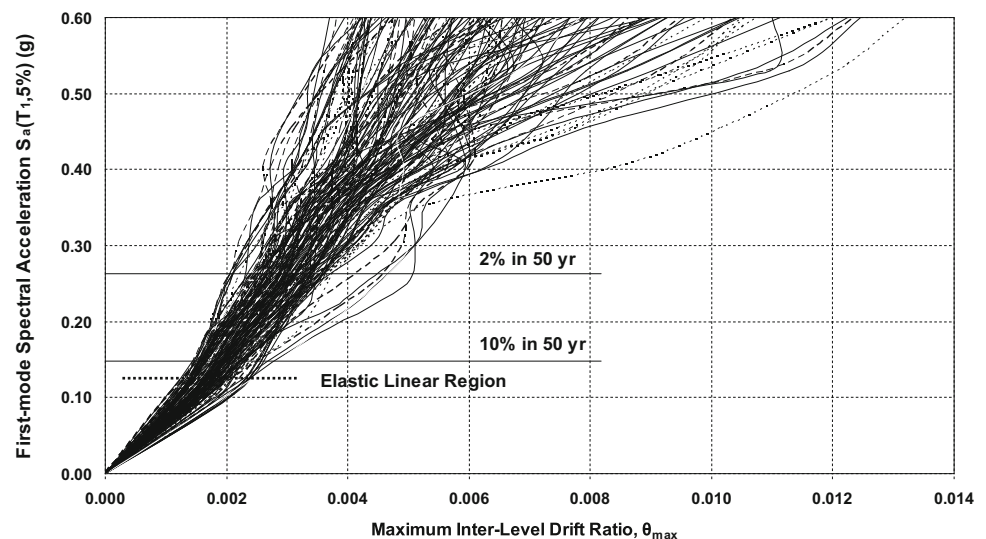
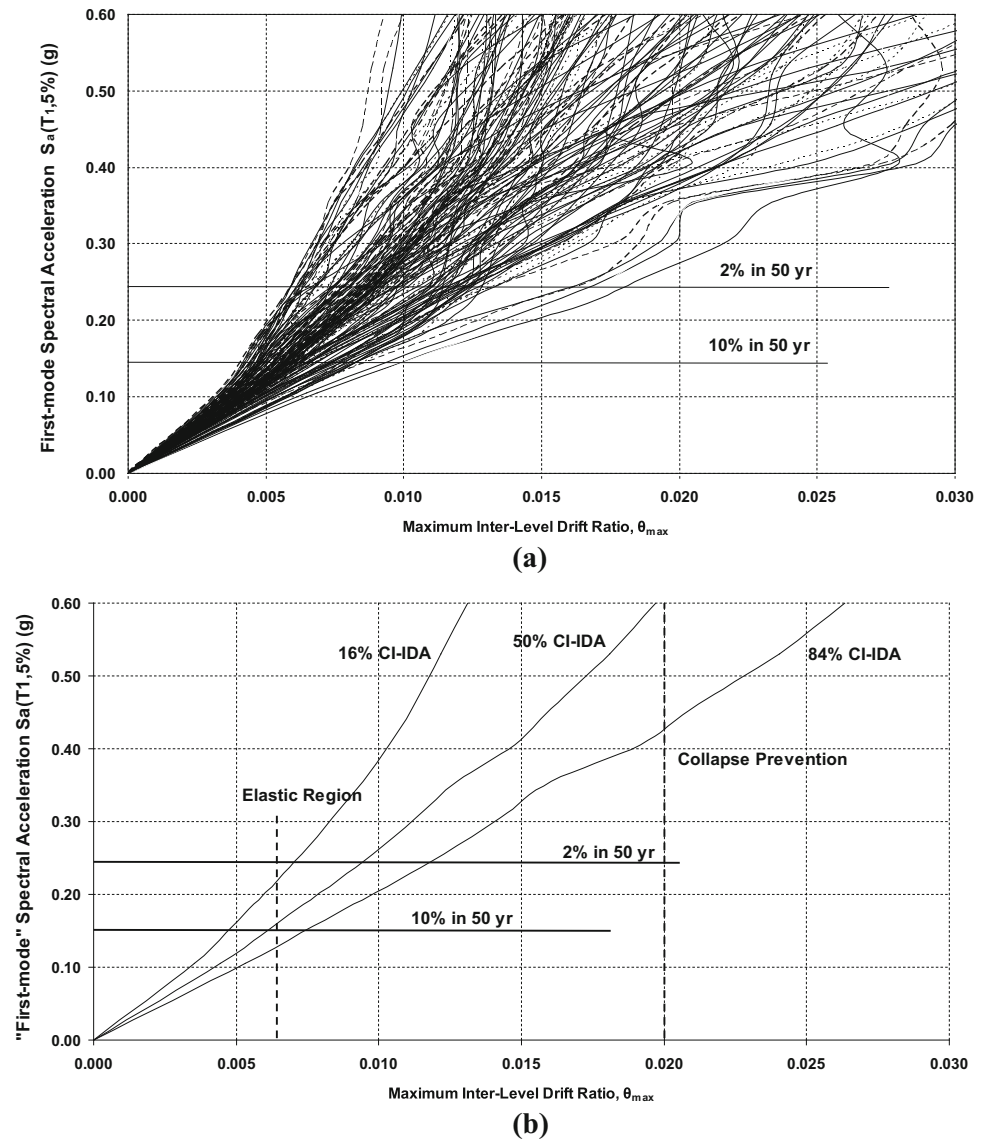
Fig. 6 The 150 raw single-record CI-IDA curves at 3rd level of the jacket

Fig. 7 Multi-record CI-IDA curves and their summaries at the working point level **a** the 150 raw single-record CI-IDA curves, **b** the 16, 50 and 84 % fractile CI-IDA curves



curves from baseline (mean) curves generated by Eqs. 6 and 7 for each shear strain value γ .

$$G/G_{max}(\gamma) = [G/G_{max}(\gamma)]_{mean} + \varepsilon_1 \times \sigma_{NG} \quad (6)$$

$$D(\gamma) = [D(\gamma)]_{mean} + \rho_{D,NG} \times \varepsilon_1 \times \sigma_D + \sigma_D \times \sqrt{(1 - \rho_{D,NG}^2)} \times \varepsilon_2 \quad (7)$$

where ε_1 and ε_2 are uncorrelated normal random variables with zero mean and unit standard deviation; $[G/G_{max}(\gamma)]_{mean}$ and $[D(\gamma)]_{mean}$ are the baseline values evaluated at strain level γ ; σ_{NG} and σ_D are the standard deviations computed from Eqs. (3) and (4) at the baseline values of $[G/G_{max}(\gamma)]_{mean}$ and $[D(\gamma)]_{mean}$, respectively, and $\rho_{D,NG}$ is the correlation coefficient between G/G_{max} and D .

Comprehensive interaction incremental dynamic analysis

CI-IDA is performed by conducting a series of nonlinear time history analyses. They require several months to accomplish the whole process. Once IM is incrementally increased in each analysis, a DM, such as global drift ratio, is monitored during each analysis. Finally, the extreme values of a DM are plotted against the corresponding value of the ground motion IM for each intensity level to produce a single-CI-IDA curve for the platform and the chosen earthquake record.

Since a single-CI-IDA curve is not able to fully capture the seismic performance, a collection of single-record CI-IDA curves is generated in numerous sets under different



Fig. 8 Level-to-level profile of θ_{\max} at different $S_a(T_1, 5\%)$ levels in all records

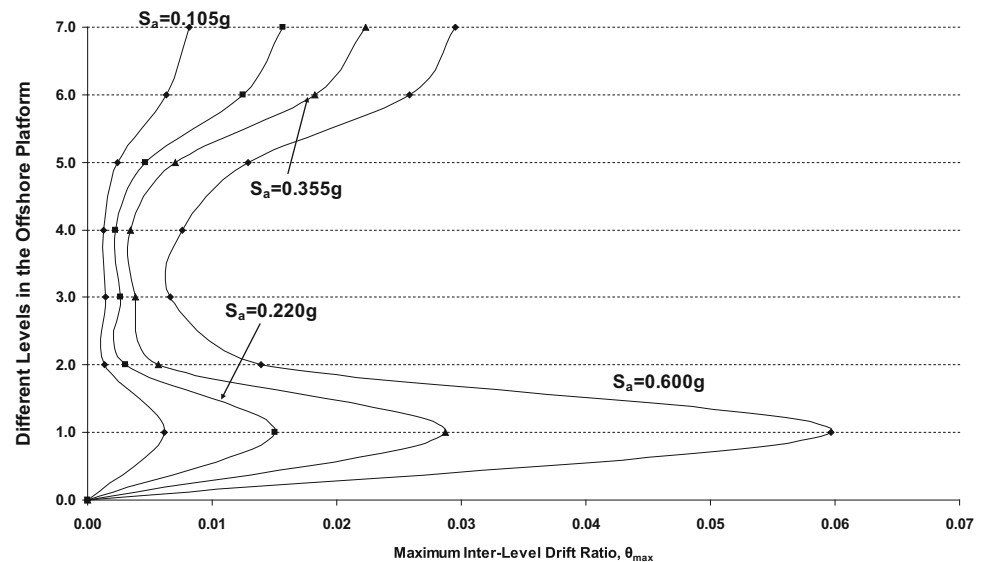
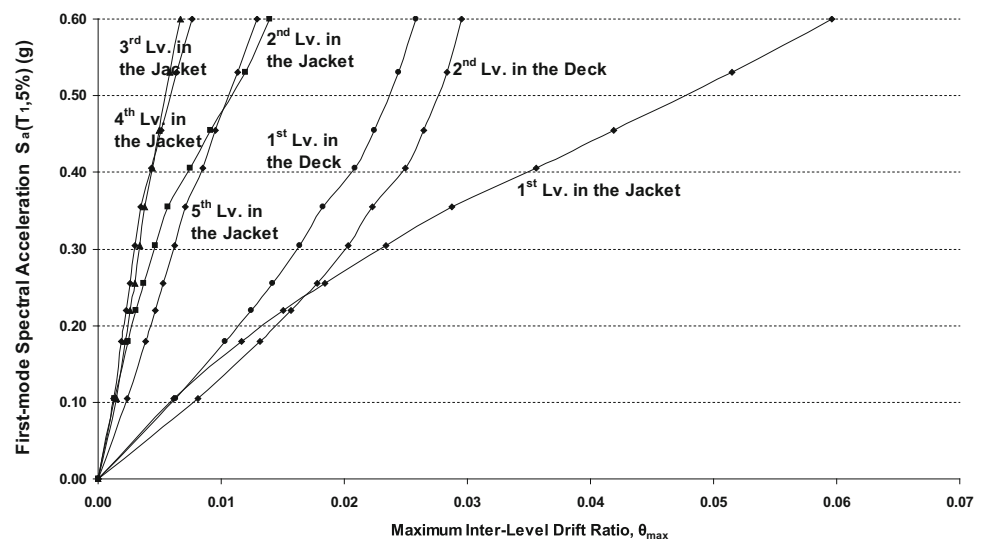


Fig. 9 The median CI-IDA curves of the offshore platform resulted from all records



accelerograms. For instance, Fig. 6 is multi-record CI-IDA curves and represents the variation of maximum inter-level drift ratio (θ_{\max}) versus the 5 %-damped first-mode spectral acceleration ($S_a(T_1, 5\%)$) at 3rd level of the jacket using the 150 raw single-record CI-IDA curves. In the Fig., the curves exhibit maximum inter-level drift ratios change from 0.12 to 0.245, 0.15 to 0.295 and 0.205 to 0.505 % for the elastic linear region, the 10 and 2 % in 50 years ground motion levels, respectively.

In multi-record CI-IDA curves, there is practical information that can be provided using appropriate summarization techniques. Therefore, the cross-sectional fractile technique was applied to generate mean, 16 and 84 % fractiles. Figure 7 shows multi-record CI-IDA curves, the 16, 50 and 84 % fractile CI-IDA curves at the working point level.

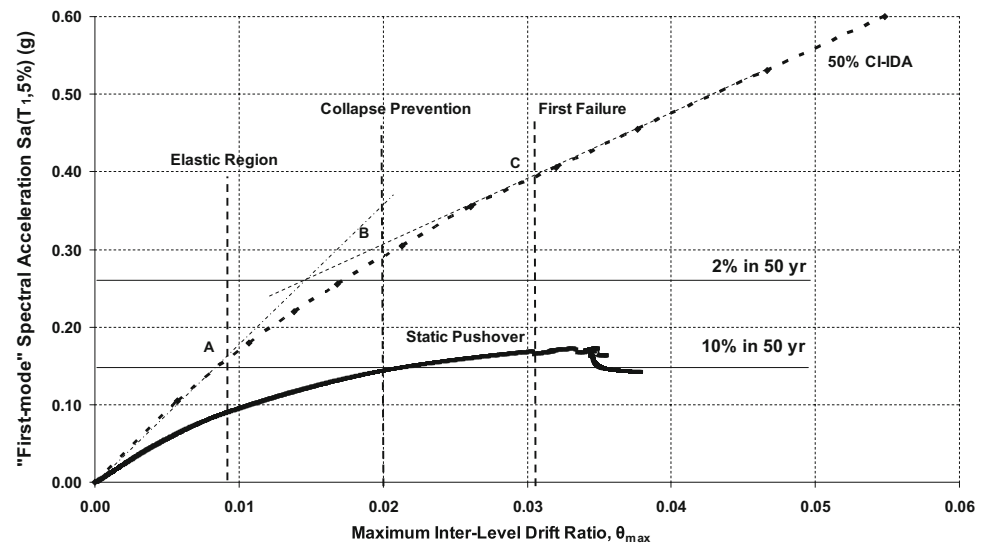
Moreover, Fig. 7 graphically defines some limit states at the working point level of the offshore platform. For instance, the 50 % CI-IDA curve presents θ_{\max} equal to 0.65, 0.60 and 0.90 % for the elastic region, the 10 and 2 % in 50 years ground motion levels, respectively.

Since the jacket-type offshore platforms are as tall structures, each level presents different drift patterns with height (Figs. 7, 8). To assess seismic performances and determine the collapse mechanisms in the offshore platforms, it is essential to quantify the seismic structural performance in the whole platform.

Figure 8 displays level-to-level profile of θ_{\max} at different $S_a(T_1, 5\%)$ levels of all selected records. The first level to the fifth level shows the levels of the jacket and the rest belongs to the deck. The figure shows there are irregularities in elevations such that the first level in the



Fig. 10 Comparison of the median CI-IDA curve (thick dash line), the simplified method curve (dash line) and the SPO curve (solid line) at the mud line level



jacket acts as a weak level relative to stronger upper levels. Also, the variation of θ_{\max} reflects the importance of addressing the effects of epistemic and aleatory uncertainties, simultaneously, to predict the seismic performance.

On the other hand, Fig. 9 shows the median variations of θ_{\max} versus $S_a(T_1, 5\%)$ in each level subjected to all records, separately. As shown in Figs. 8 and 9, while the mud line level acts as a fuse in the whole platform, there is a relative rigid core in the second, third and fourth levels of the jacket and gradually the levels of damage increase in the fifth level of the jacket and the first and second levels of the deck. It should be mentioned that the mud line level elevation is very close to the first level of the jacket; therefore, they have very similar seismic performances.

Comparison of the simplified method and CI-IDA

In this study, the simplified and the CI-IDA methods were performed in the same offshore platform. Figure 10 represents comparison of the median CI-IDA curve (thick dash line), the simplified method curve (dash line) and the SPO curve (solid line) at the mud line level.

As per the results, the maximum inter-level drift ratio at the 10 % in 50 years ground motion level resulted from the simplified method and mean CI-IDA is about 0.8 % while at the 2 % in 50 years ground motion level is 1.4 and 1.7 %, respectively.

Moreover, θ_{\max} is equal to 0.9 % in the elastic linear region of the simplified method and mean CI-IDA curves and ends at $S_a(T_1, 5\%) \approx 0.16$ g and 0.15 g, respectively. Furthermore, the CP in the simplified method and mean CI-IDA occurs at $S_a(T_1, 5\%) \approx 0.31$ g and 0.28 g,

respectively. The results indicate that the simplified method is a straightforward approach that is able to predict different limit states with reasonable accuracy.

Conclusions

In this paper, a simplified method has been proposed. It can approximate the seismic demands of jacket-type offshore platforms from elasticity to global collapse. The method is based on the static pushover (SPO) and is able to predict the 50 % fractile of the comprehensive interaction incremental dynamic analysis (CI-IDA) curve for the platform with reasonable accuracy. Since CI-IDA is a computer-intensive procedure, the simplified method creates a direct connection between the results of SPO and CI-IDA analyses. It is a valuable tool that is very attractive for the engineer users and reduces the analysis time from 48 h to only several minutes.

On the other hand, seismic evaluation of the platform indicates that it is possible to take into account the whole platform into three parts. The first part is the mud line and first levels that experience the collapse mechanism at high levels of ground motion intensity, while the second, third and fourth levels of the jacket have relative rigid core and finally the fifth level of the jacket and the first and second levels of the deck sustain some damage.

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